The depth of the chamber can now be selected based upon usual considerations of soil condition, and land cost, etc., although, as will be seen later, shallower depths than usual are preferable. The remaining problem is to ensure that the required slope is obtained. The required slope is calculated from Equation 1. The slope in open channel flow can be calculated by Mannings' equation:

$$\text{Slope} = (\text{Velocity})^2 \left[\frac{n}{1.49}\right]^2 \left[\frac{1}{R}\right]^{4/3}, \qquad \text{(Eq. 2)}$$

where "n" is a factor relating to the obstruction to flow of obstacles at walls and within the channel. This factor is historically called a "roughness factor" and the numerical value found in hydraulics handbooks is 0.011 for steel or neat concrete and 0.03 for the situation where corrugated metal froms the wall of a channel whose width is several hundred times the corrugation height. For our purpose, this could be considered a turbulence promotion factor. Work is in progress to determine the effective turbulence promotion effect of corrugated baffles in narrow passages where we believe it to be at least twice the 0.03 value given above. The effect for other configurations is being studied as well. The term "roughness factor" will be used until a more appropriate term is coined.

The hydraulic radius "R" is the ratio of the cross-sectional area of the passage in ft^2 to the wetted perimeter in feet.

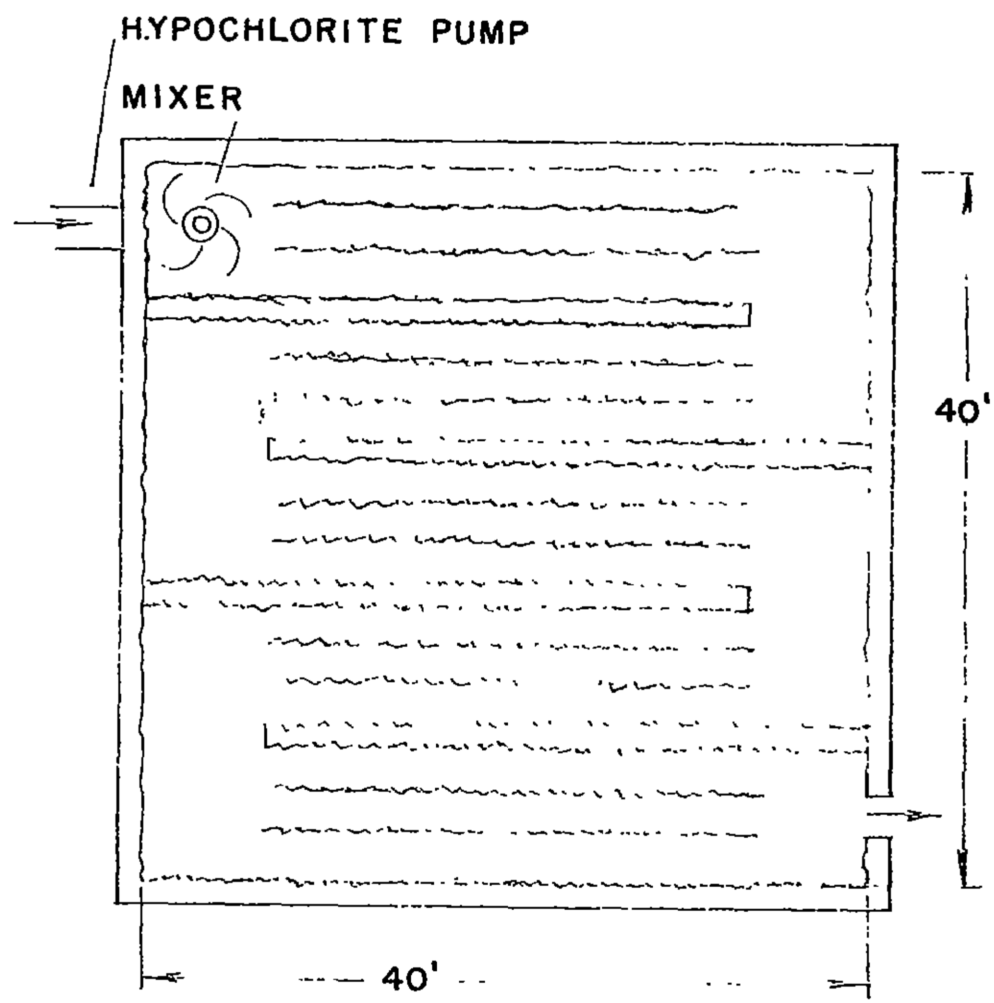
Since the velocity has been fixed and the required slope calculated, only the roughness factor relating to the type of wall and/or baffle surface and the hydraulic radius relating to the wall area parallel to the flow path can be governed by the designer.

The combined effect of these two variables is calculated from Equation 2.

For illustration in Figure 3, corrugated baffles parallel to the path are shown. In this simplified sketch, the significant dimensions are shown. The passage width is fixed by the selected velocity and channel depth. The number of the parallel baffles inserted determines the hydraulic radius. The roughness factor is determined primarily by the surface of the baffle material selected.

In spite of the undeveloped state of this design scheme, we were able to produce a chamber within 6% (9,400) of the design target (10,000), on our first attempt. Also, additional baffles can be easily inserted at a later date if required.

This design scheme yields considerable insight to the evaluation of the performance of existing and future contact chambers. The disinfection performance has been shown to be a function of the GT parameter. In conventional chambers the outlet weir is located near the design rate water level so that the water volume is nearly constant at all flow rates. As can be seen by Equations 1 and 2, the G varies as the $(\text{velocity})^{1.5}$. With constant liquid level, the T varies as (1/velocity), thus the GT parameter will vary as the $(\text{velocity})^{.5}$ or with $(\text{flow rate})^{.5}$. This poorer performance at reduced flow rate would escape attention under relatively constant rate conditions in a sewage plant. However, under the widely variable rate conditions met in



TOP VIEW OF A 92 CFS (60 MGD)
INTENSLY MIXED CHLORINE CONTACT CHAMBER
(AVG. WATER DEPTH 7') RESIDENCE TIME 120 SECONDS

FIGURE 3

combined sewer overflow service, it must be considered. The use of a Sutro weir has been proposed to maintain a constant velocity at all flow rates.

A 92 cfs (60 mgd) Intensity Mixed Chlorine Contact Chamber has been designed. This chamber was designed to follow a microstrainer facility with 46 cfs treatment capacity and an additional 46 cfs bypass capacity. The chlorine contact chamber was designed to have 120 seconds residence time at the 92 cfs rate and, since a Sutro weir is used the residence time at less than the 92 cfs rate will be about 120 seconds also. The velocity is 1.5 ft/sec and the amount of baffling and its configuration is such to yield a velocity gradient G of 40, as in our pilot plant.

The chamber is 40' by 40' and has an average liquid depth of 7' at maximum flow. Internal walls form a labyrinthine-like passage of 8' in width and produce a velocity of 1.5 ft/sec. The internal walls are faced with a commercially available corrugated asbestos siding having 1-1/2" deep corrugations.

Two additional corrugated panels are mounted as parallel baffles in the channels forming 32 inch wide passages. The baffles extend from liquid level to within a foot of the floor. Ideally the floor would be similiarly corrugated, but this is not necessary. The head loss through the chamber at peak flow is about 8 inches. (See Figure 3)

The inlet to the chamber is equipped with a 3 hp mixer sweeping an 8' x 8' section of the channel (about 5 sec residence time). A mixer of this horsepower should be able to impart 1 hydraulic horsepower to the water to enduce a mixing intensity of about 200 sec^{-1} in this 450 ft^3 volume, which should be adequate for thoroughly mixing the chlorine chemicals. Such a provision for mixing of chemicals is incomparably superior to the methods usually used in sewage plants. The mixer should be of such a type that it can operate at varying water levels from 7' down to 1'.

The outlet of the chamber should be fitted with a relatively narrow outlet weir placed as low as the available outfall head will allow, preferably at the bottom. Further, the outlet weir should be of the Sutro type to maintain the velocity in the chamber, at less than peak rate, as near the peak rate velocity as possible. A Sutro weir at the bottom will maintain peak rate velocity at all flow-thru rates. In the event the allowable outfall head will not permit placing the weir at the bottom, a small pump must be provided to empty the chamber at the end of the storm.

The installed cost of such a chamber has been calculated to be about \$53,000 (in 1969 dollars) less the cost of land, engineering and profit (1). It is difficult to compare costs developed by different estimators. However, this cost can be compared to the data developed by Smith (15) of \$25,000 for an \$11,000 ft^3 basin, which is the volume of the basin described above. Also, it can be compared to Smith's estimate of \$90,000 for the 81,000 ft^3 chamber required to provide 15 minutes residence for 60 mgd in a conventional chamber.

The inherent advantage of increased turbulence economically induced in this type of installation to enhance reaction rates can be used in many situations. An obvious example would be to use it in chlorine contact chambers at sewage plants with savings in construction cost, land, and the advantage of high virus kill and reliable bacteria kill.

ACKNOWLEDGEMENT

This work was conducted with the City of Philadelphia in two phases, (1) under a contract from the Environmental Protection Agency to the Cochrane Division of the Crane Co., and (2) under an EPA grant to the City of Philadelphia. The efforts of City personnel were under the general direction of Carmen Guarino, Water Commissioner, with William Wankoff and M. Lazanoff, serving as Project Director and Laboratory Director. J. Radzuil headed the City's R and D Department who also lent valuable assistance.

The assistance and guidance of these people are gratefully acknowledged.

The overall guidance and helpful advice of Richard Field, Project Officer, EPA, Edision, New Jersey, were most valuable.

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SECTION IX

THE SWIRL CONCENTRATOR AS A COMBINED SEWER OVERFLOW REGULATOR

by

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A report by the American Public Works Association published in 1970 gave the results of a study of combined sewer overflow regulator facilities. Design, performance and operation and maintenance experiences from the United States and Canada, and in selected foreign countries were reported. It was evident that North American practice has emphasized the design of regulators simply as flow splitters, dividing the quantity of combined sewage to be directed to the treatment facitilities, and the overflow to receiving waters. Little consideration was given to improving the quality of the overflow wastewater.

Using hydraulic laboratory tests and mathematical modeling strongly we have determined that it is possible to remove significant portions of settleable and floatable solids from combined sewage overflows by using a swirl concentrator. The practical, simple structure has the advantages of low capital cost; absence of primary mechanical parts should reduce maintenance problems; and construction largely with inert material should minimize corrosion. Operation of the facility is automatically induced by the inflowing combined sewage so that operating problems normal to dynamic regulators such as clogging will be very infrequent.

The device, as developed, consists of a circular channel in which rotary motion of the sewage is induced by the kinetic energy of the sewage entering the chamber. Flow to the treatment plant is deflected and discharges through an orifice called the foul sewer outlet, located at the bottom and near the center of the chamber. Excess flow in storm periods discharges over a circular weir around the center of the tank and is conveyed to storage treatment devices as required or to receiving waters. The concept is that the rotary motion causes the sewage to follow along a spiral path through the circular chamber.

A free surface vortex was eliminated by using a flow deflector, preventing flow completing its first revolution in the chamber from merging with inlet flow. Some rotational movement remains, but in the form of a gentle swirl, so that water entering the chamber from the inlet pipe is slowed down and diffused with very little turbulence. The particles entering the basin spread over the full cross section of the channel and settle rapidly. Solids are entrained along the bottom, around the chamber, and are concentrated at the foul sewer outlet.

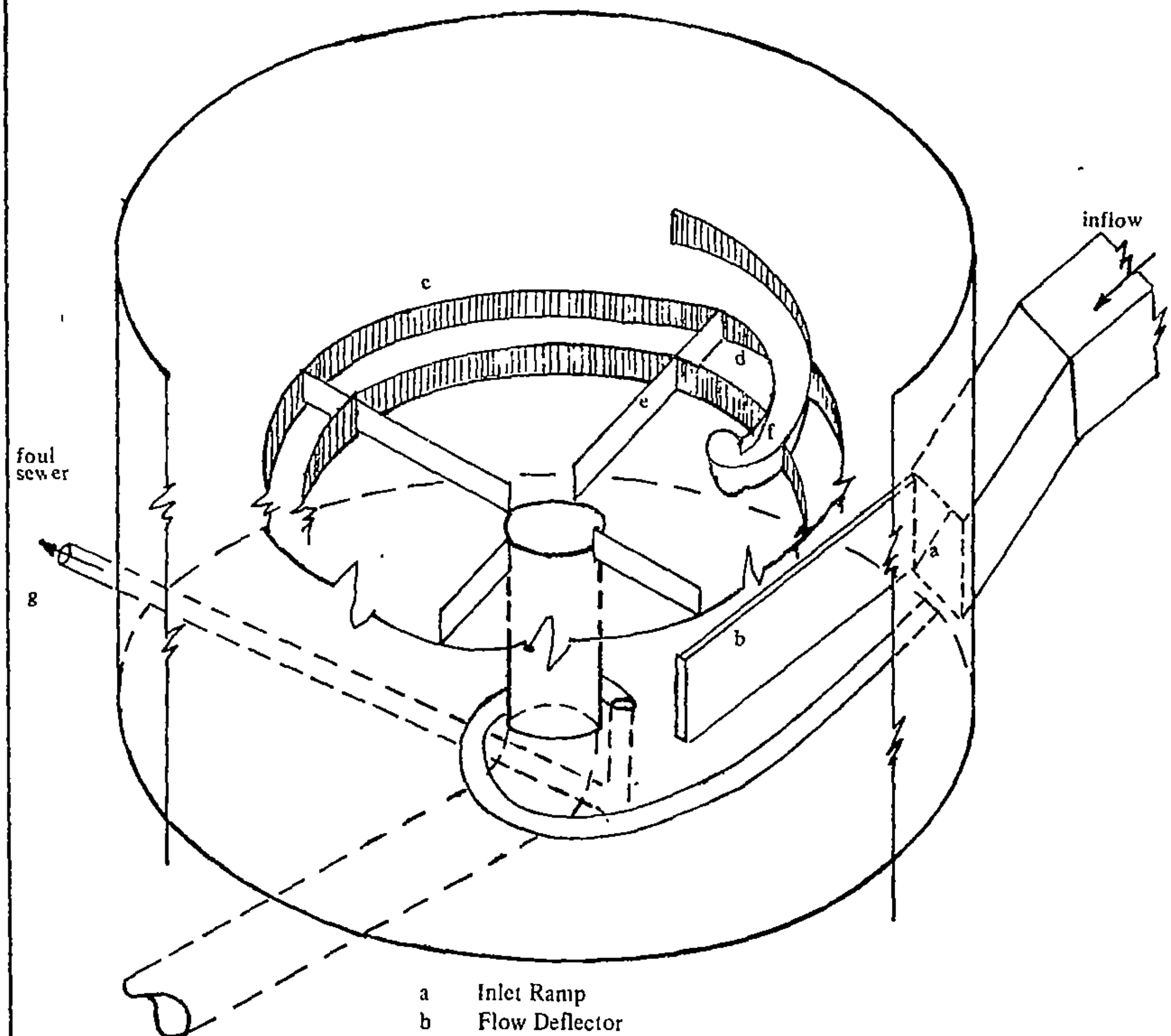
Figure 1, Isometric View of Swirl Concentrator, depicts the final hydraulic model layout showing details such as the floatables trap, foul outlet and floor gutters.

The swirl concentrator may have practical applications as a degritter, or grit removal device for sanitary sewage flows or separate storm water discharges of urban runoff waters. It may have capabilities for the clarification of sanitary sewage in treatment plants, in the form of primary settling or, possibly, final settling chambers. It might be used for concentrating, thickening, or elutriating sewage sludges. It may be serviceable in the separation, concentration and recycling of certain industrial waste waters, such as pulp and paper wastes or food processing wastes, with reuse of concentrated solids and recirculation of clarified overflow waters in industrial processing closed circuit systems.

In water purification practices, it may find feasible applications in chemical mixing, coagulation and clarification of raw water. Other uses may prove to be realistic and workable.

Complete reports describing the hydraulic laboratory study and the mathematical modeling are included in the report EPA R2-72-008, September 1972, published by USEPA. The body of the report details the basis of the assumptions used to establish the character and amount of flow to be treated and the design of a swirl concentrator based upon the hydraulic and mathematical studies.

Although the study was performed for the City of Lancaster, Pennsylvania, with a specific point of application defined, all work was accomplished in a manner which allows ready translation



a Inlet Ramp
b Flow Deflector
c Scum Ring
d Overflow Weir and Weir Plate
e Spoilers
f Floatables Trap
g Foul Sewer Outlet
h Floor Gutters

FIGURE 1
ISOMETRIC VIEW OF SWIRL CONCENTRATOR

application of the results to conditions which might be found at other installations and for other purposes.

Consideration of the use of a swirl concentrator as a combined sewer overflow regulator facility requires an evaluation of many factors which include:

1. hydraulic head differential between the collector and interceptor sewers and head available in collector sewer to allow insystem storage;
2. hydraulic capacity of collector sewer;
3. design flow;
4. dry-weather flow and capacity of interceptor sewer; and
5. amount and character of settleable solids.

Although many of these items have been mentioned in the preceding sections of the report, the importance of each will be highlighted in order to emphasize the importance of each point in a preliminary evaluation of the use of the swirl concentrator.

Hydraulic Head Differential. There must be sufficient hydraulic head available to allow dry-weather flows to pass through the facility and remain in the channel. The total head required for operation is shown in Figure 2, Hydraulic Head Requirements. Determination of the maximum elevation in the collector sewer that can be utilized for insystem storage and the differential elevation between the collector and interceptor sewers is the total available head.

The head required will vary directly with flow and the outlet losses in the foul sewer.

If sufficient head is not available to operate the foul sewer discharge by gravity, an economic evaluation would be necessary to determine the value of either pumping the foul sewer outflow continuously, or pumping the foul flow during storm conditions and bypassing the swirl concentrator during dry-weather conditions, perhaps with a fluidic regulator.

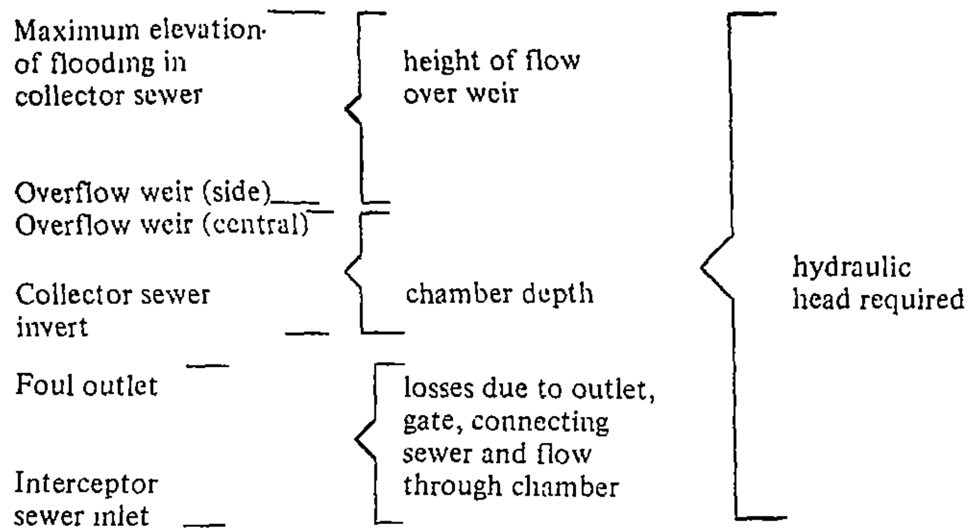


FIGURE 2
HYDRAULIC HEAD REQUIREMENTS

<u>Hydraulic Capacity of Collector Sewer System</u>. The facility must be designed to handle the total flow which might be delivered by the collector system. Thus a study of the drainage area must be made to determine the limiting grade and pipe sizes which control the quantity of flow. Solids removal from a peak flowrate may not be required. If the chamber is not designed for such maximum flows, however, velocity energies which could be developed at such full flow conditions should be avoided by providing a bypass in the form of a side overflow weir.

<u>Design Flow.</u> Selection of the design flow for sizing the chamber should be accomplished on the basis of a complete hydrological study to determine frequency and amount of precipitation which can be anticipated as well as runoff hydrographs. Computer models such as developed by the University of Florida for USEPA can be of assistance in determining the solids load which may be associated with various amounts and intensity of precipitation. Provision of maximum solids removal for a two-year frequency storm for the Lancaster, Pennsylvania, project was made on the basis of engineering judgment and an evaluation of local receiving water conditions. As the cost of construction will increase in direct proportion to design flow, an economical evaluation should generally be used to select the flow capacity. The efficiency curve

for the facility is rather flat over a wide range of flows, resulting in perhaps large increases in cost for marginal improvements in efficiency.

A major constraint in selecting large design flows is the anticipated shoaling problems of solids at low flow rates in large facilities. Self cleaning is enhanced by reduced diameters. This consideration may make it desirable to design for lower flows, particularly where some form of overflow treatment is to be provided. Again the computer model can be used to determine the magnitude of the solids carry-over problem to the secondary device.

A third consideration is the maintenance of low-inflow velocities, with turbulence minimized. At the design flow the inflow velocity should be in the range of three to five fps. The inflow velocity may require reduction by enlarged pipe sections or other means to achieve this rate.

Dry Weather Flow and Capacity of Interceptor Sewer. Sizing of the foul sewer, the foul outlet and the gutter depend upon a determination of the dry-weather flow in addition, the capacity of the interceptor sewer to handle the foul flow must be known. The foul sewer must be large enough to maintain and not be subject to blockage--- usually a minimum 12-inch diamter. However, the head on the outlet during overflow conditions will allow considerable variations in the foul discharge if it is not controlled.

The efficiency of the chamber is affected by the ratio of foul flow to overflow---although there appears to be a broad operating range over which reasonable removal efficiencies can be maintained.

Maximum advantage should be taken of capacity in the interceptor system, particularly during the period when the chamber is being drawn down. Thus, sensing of the flow in the interceptor and the use of a control gate on the foul sewer appear desirable to obtain maximum results from the use of the chamber.

Amount of Character of Settleable Solids. The sewer system must provide capacity to handle the increase in settleable solids which will be captured from the combined sewer overflow and discharged to the treatment plant. In the case of Lancaster, Pennsylvania, this could amount to more than a ton of solids from one device in a very short period of time. Additional grit removal and sludge processing equipment may be necessary. Should the foul flow be pumped, sumps and pumps should be designed to handle the anticipated high solids content.

If the settleable solids which can be anticipated in the combined sewer overflow can be defined by the amount, specific gravity, and particle size, the mathematical and the hydraulic model may be used to determine the size of the chamber required to achieve desired levels of solids removal. Ordinarily this will not be feasible and the flow criteria developed by the hydraulic model will be used to design the facility and predict removal efficiencies.

In order to evaluate the efficiency of the chamber, facilities should be provided for sampling the inflow, foul sewer flow and overflow. Settleable solids should be delineated in all of these flows. The quantity of inflow and foul sewer flow should also be measured. Difficulties in obtaining representative samples from any of the flows may make evaluation difficult. However, the treatment plant or combined sewer overflow treatment facility , if used, should provide an excellent means of making a gross evaluation into the effectiveness of the chamber.

Provision of a means to measure the depth of flow over the weir should act to give a reliable measurement of the flow when added to the quantity of flow to the foul sewer.

Data from many full-scale operations, operating with various flow conditions and solid loadings will be necessary to properly evaluate the usefulness of the swirl concentrator as a combined sewer overflow regulator.

Cost of Facility. The cost of construction of the swirl concentrator will vary with the length of inlet pipe which must be reconstructed, the depth of the chamber and the nature of the material to be excavated, the need for a roof, and the general site conditions under which the work will be conducted. The materials of construction will usually be concrete and steel and elaborate form work will not be required.

For the Lancaster, Pennsylvania, application where a 36 foot diameter chamber in limestone is contemplated, the preliminary estimate of cost was $100,000 in 1972 costs. This cost estimate included a roof, foul sewer outlet control and a wash-down system. Site construction problems are minimized in as much as the construction will be off of the street right-of-way.

SECTION X

THE EPA STORMWATER MANAGEMENT MODEL

A CURRENT OVERVIEW

by

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1. INTRODUCTION

A. COMBINED AND STORM SEWER OVERFLOWS

An enormous pollution load is placed on streams and other receiving waters by combined and separate storm sewer overflows. It has been estimated that the total pounds of pollutants (BOD and suspended solids) contributed yearly to receiving waters by such overflows is of the same order of magnitude as that released by all secondary sewage treatment facilities (Gameson and Davidson, 1964; Field and Struzeski, 1972). The Environmental Protection Agency (EPA) has recognized this problem and led and coordinated efforts to develop and demonstrate pollution abatement procedures (Field and Struzeski, 1972). These procedures include not only improved treatment and storage facilities, but also possibilities for upstream abatement alternatives such as rooftops and parking lot retention, increased infiltration, improved street sweeping, retention basins and catchbasin cleaning or removal. The complexities and costs of proposed abatement procedures require that care and effort be expended by municipalities and others charged with decision making for the solution of these problems.

B. THE STORM WATER MANAGEMENT MODEL

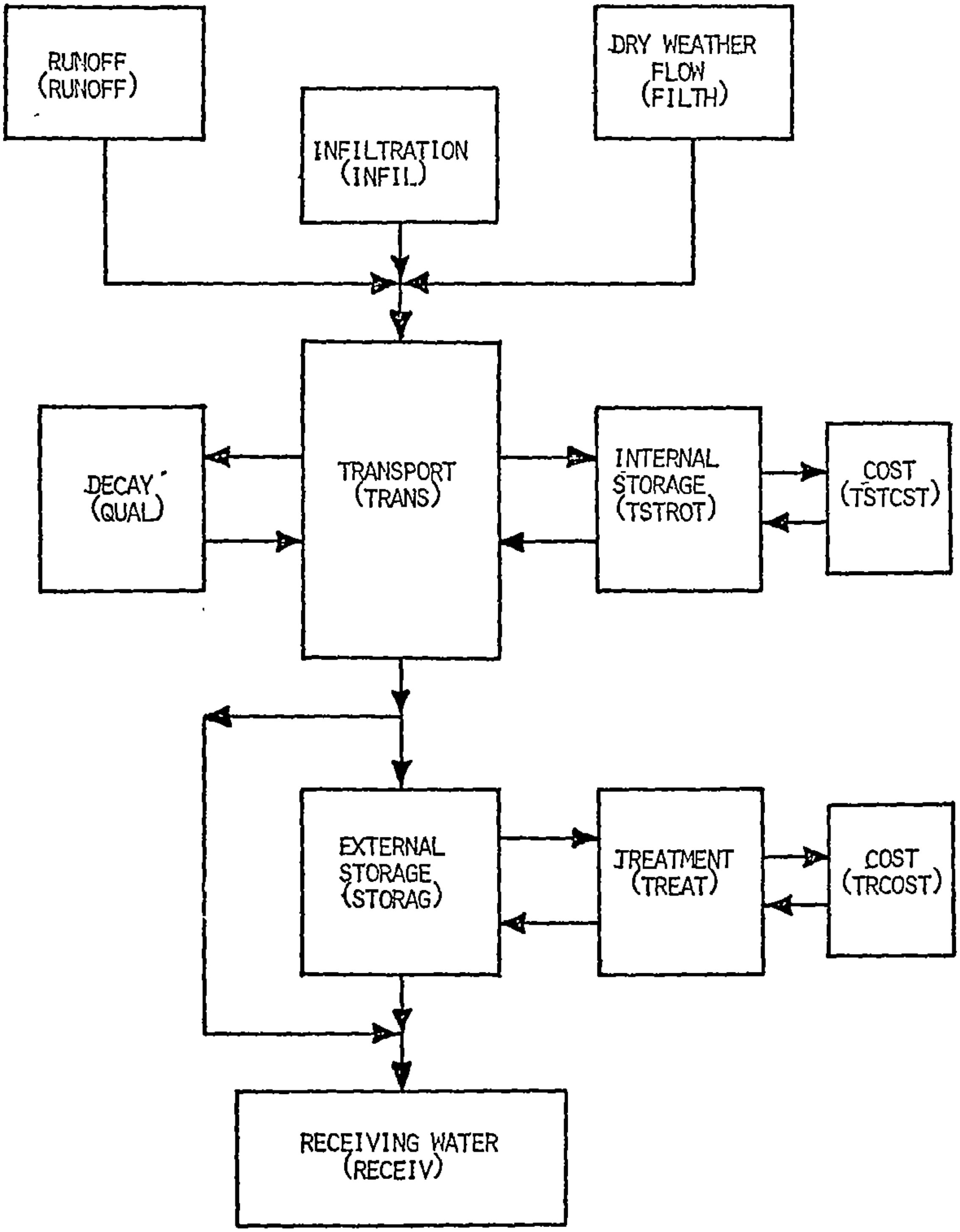
It was recognized that an invaluable tool to decision makers would be a comprehensive mathematical computer simulation program that would accurately model quantity (flow) and quality (concentrations) during the total urban rainfall-runoff process. This model would not only provide an accurate representation of the physical system, but also provide an opportunity to determine the effect of proposed pollution abatement procedures. Alternatives could then be tested on the model and least cost solutions could be developed.

As a result, the University of Florida (UF), Metcalf and Eddy, Inc., Engineers (ME) and Water Resources Engineers (WRE) were awarded a joint contract for the development, demonstration and verification of the Storm Water Management Model (SWMM). The resulting model, completed in October, 1970, has been documented (EPA, 1971a, b, c, d) and is presently being used by a variety of consulting firms and universities.

The present SWMM is descriptive in nature and will model most urban configurations encompassing rainfall, runoff, drainage, storage-treatment, and receiving waters. The major components of the SWMM are illustrated in Figure 1-1. However, it does not define nor determine any decisions for the system or consider alternative methods for efficient economic comparisons.

C. DECISION MAKING

In recognition of the need for improved decision making capabilities, the University of Florida submitted a proposal to EPA titled "A Decision Making Model for the Management of Storm Water Pollution Control" in which it was intended to provide a systematic procedure which could be applied to a wide variety of specific circumstances in support of intelligent management decisions. The work required to obtain a least cost solution would be considerably



Note: Subroutine names are shown in parentheses.

Figure 1-1
Overview of Model Structure

reduced by means of determining the origin of the most severe pollution load, consideration of all upstream and downstream pollution abatement procedures and associated costs, and through the possible use of mathematical optimization techniques.

The project was funded as part of an EPA Demonstration Grant to Lancaster, Pennsylvania (Federal Grant No. 11023GSC), in which an underground "silo," a swirl concentrator and a microstrainer were to be installed at the outfall of the Stevens Avenue Drainage District to control overflow into the Conestoga Creek (details are presented in the next section).

Results of the decision-making methodology and other aspects of the research have recently been formulated (Heaney and Huber, 1973). Decision-making for urban storm water management is presented in the broader context of urban water resources management. Pollution sources and control options are inventoried and accompanied by economic data. Performance standards are considered and the importance of automobile-related facilities (*e.g.*, streets, parking lots, curbs and gutters) as contributors to storm water pollution and quantity is emphasized. Finally, a linear programming and game theory approach is used to develop efficient and equitable control strategies.

This paper presents an overview of the SWMM by illustrating its use in Lancaster; the following section is taken from the Final Report (Heaney and Huber, 1973) from which other details are available. Major revisions to the Model have been made to include urban erosion prediction, modeling of new treatment devices and biological treatment facilities, monitoring of significant pollution sources, flexibility in modeling new areas, new and improved cost functions for treatment and storage options and a modest hydraulic design capability as well as minor programming changes and slight format revisions. The SWMM has proven to be a useful and economical tool in the assessment of urban storm water problems. Individual runs described in the following section, for instance, could be accomplished using less than three minutes of CPU time on the IBM 370/165 at the University of Florida Computing Center, for a Runoff-Transport-Storage/Treatment-Receiving simulation. Although computational changes vary, they are well within reasonable bounds.

2. TESTING IN LANCASTER, PENNSYLVANIA

The City of Lancaster, Pennsylvania, population 79,500, is situated in a drainage area of about 8.24 square miles (5,274 acres). The receiving stream in the Lancaster area is the Conestoga Creek which drains an area of approximately 473 square miles into the Susquehanna River. The average flow is 387 cubic feet per second with a maximum recorded flow of 22,800 cubic feet per second.

There are two sewage treatment plants within the city, both of which discharge into the Conestoga Creek. The North Plant with a capacity of 10 mgd serves a population of 36,000 people, and the South Plant recently expanded from 6 mgd to 12 mgd and is designed to serve 69,000 people. Both plants provide secondary treatment. About one third of the flow to the North Plant is derived from areas with separate sewers outside the city serving an estimated population of 17,500 people and some industries. The remaining two thirds of the sewage flow to the North Plant is derived from the combined sewers serving the north part of the city plus about 250 suburban acres estimated to have 18,500 people and many water-using industries. In addition, most of the year the water table is high resulting in considerable infiltration. An overflow line diverts excess flow to the Conestoga during wet weather. The North Plant drainage area is estimated at 3.72 square miles.

The South Plant is designed to handle a population of 34,500 served by combined sewers and, in addition, up to an approximately equal amount from separated sewers throughout the surrounding area. The South Plant drainage area encompasses 4.52 square miles and is comprised of four districts. Stevens Avenue district which is the subject of EPA demonstration grant is one of the four districts connected to the South Plant. Three of the districts, including Stevens Avenue, pump the sewage from a receiving station within the district to the South Plant. All locations have overflow arrangements that discharge into the Conestoga Creek when the capacity of the system is exceeded.

The total drainage area of the Stevens Avenue district is 227 acres which, while only about 4.3% of the total Lancaster drainage area served by North and South treatment plants, is 17% of the drainage area designed to flow into the South Plant from combined sewers. The population within the Stevens Avenue district is estimated at 3,900. Figure 2-1 illustrates various drainage districts within the city.

1. DEMONSTRATION GRANT DESCRIPTION

In order to remedy the situation resulting from combined sewer overflows, the City of Lancaster decided to explore means other than sewer separation. Construction of several underground silos at various locations within the city is contemplated for retention of overflow during wet periods and subsequent pumping to the treatment plants during low flow periods.

Stevens Avenue district was selected as the demonstration site for evaluation of the effectiveness of a silo in combating combined

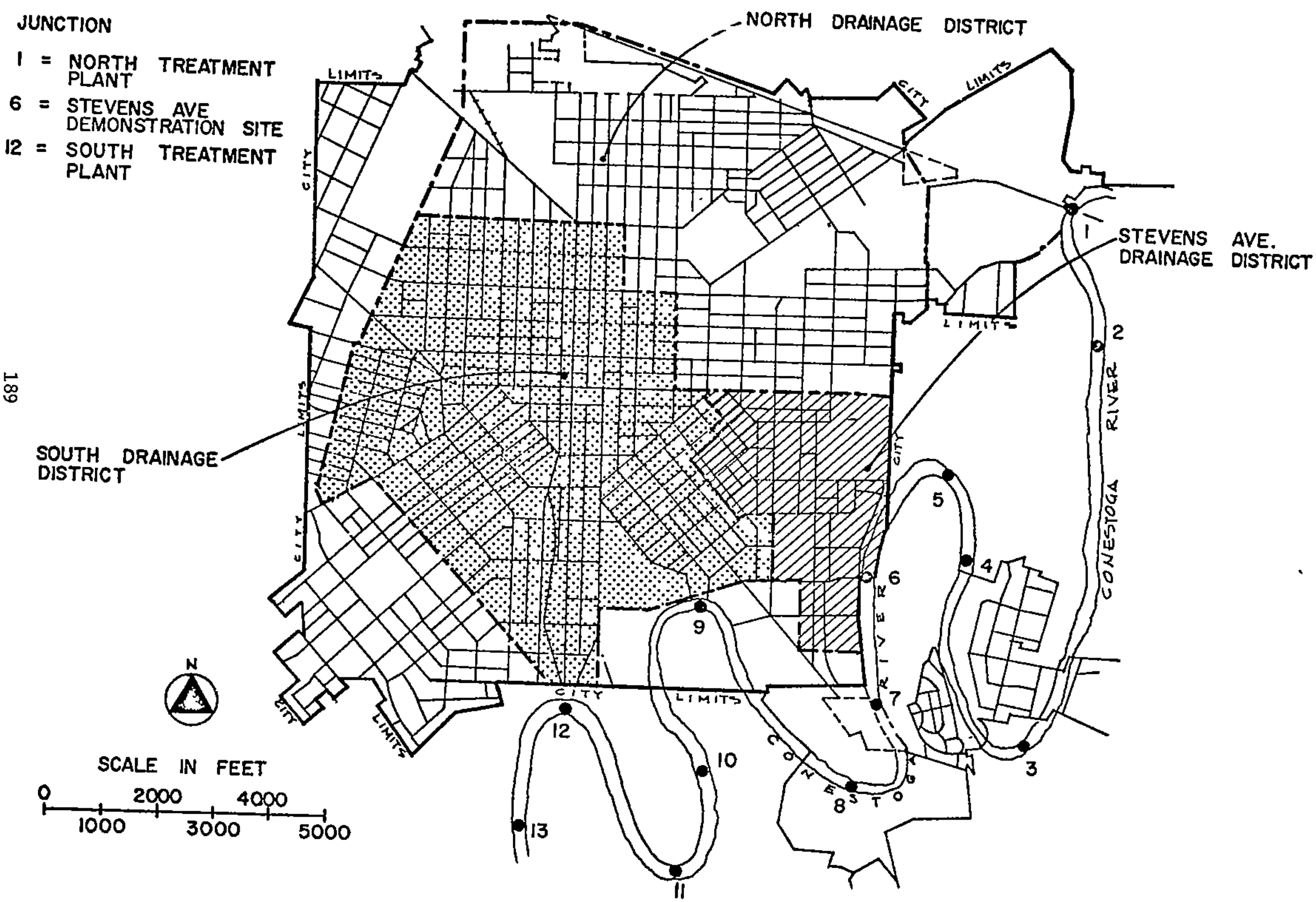


Figure 2-1
Drainage Districts of Lancaster, Pennsylvania and Numbering System for Receiving Junctions.

sewer overflows. The sewer layout for Stevens Avenue district is shown in Figure 2-2. During normal dry weather periods, the dry weather flow is pumped to the South treatment plant. During wet periods, when the incoming flow to the pump station exceeds the capacity of the station, the overflow discharges directly into the Conestoga Creek through a 60 inch sewer located at point 6 on Figure 2-1.

The City of Lancaster also authorized APWA to develop design parameters for a full-scale swirl concentrator for removal of solids prior to the retention of flow in the underground silo. Location of the demonstration site is shown in Figure 2-2. A flow diagram of the proposed swirl concentrator-silo treatment is presented in Figure 2-3. In order to fully evaluate this treatment the city decided to include chlorination and microstraining as a part of this demonstration project. The capacity of the silo is expected to be 160,000 cf.

The tasks assigned to the University of Florida were as follows:

1) Conduct further verification and testing of the Storm Water Management Model based on active overflow measurements on selected storm events and to make refinements to the Model;

2) Provide results of simulations to the APWA in order for it to develop design criteria and sizing of the swirl concentrator;

3) Simulate the effect of the swirl concentrator-underground silo treatment; and

4) Simulate the effect of combined sewer overflow from the entire city to the Conestoga Creek.

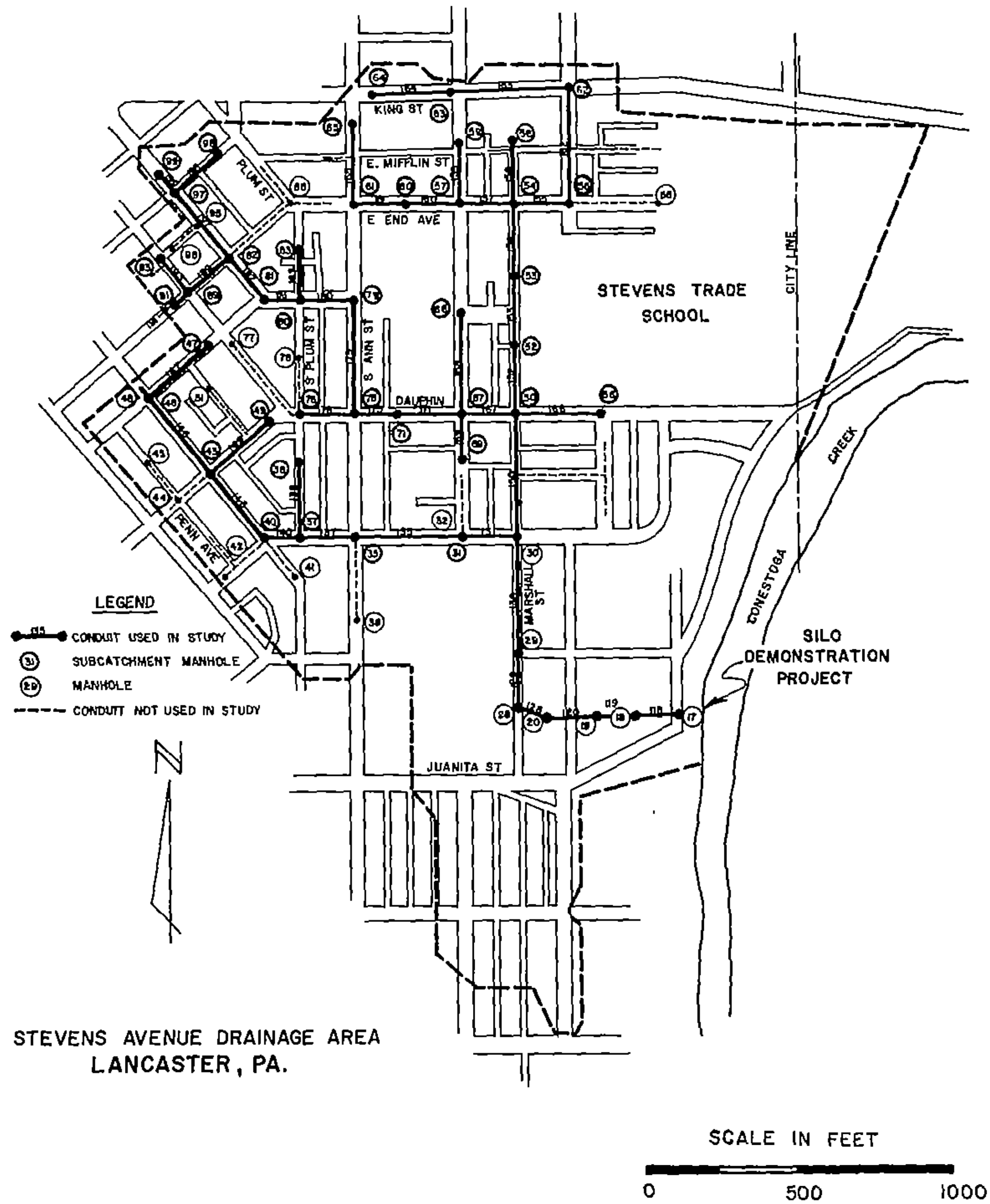
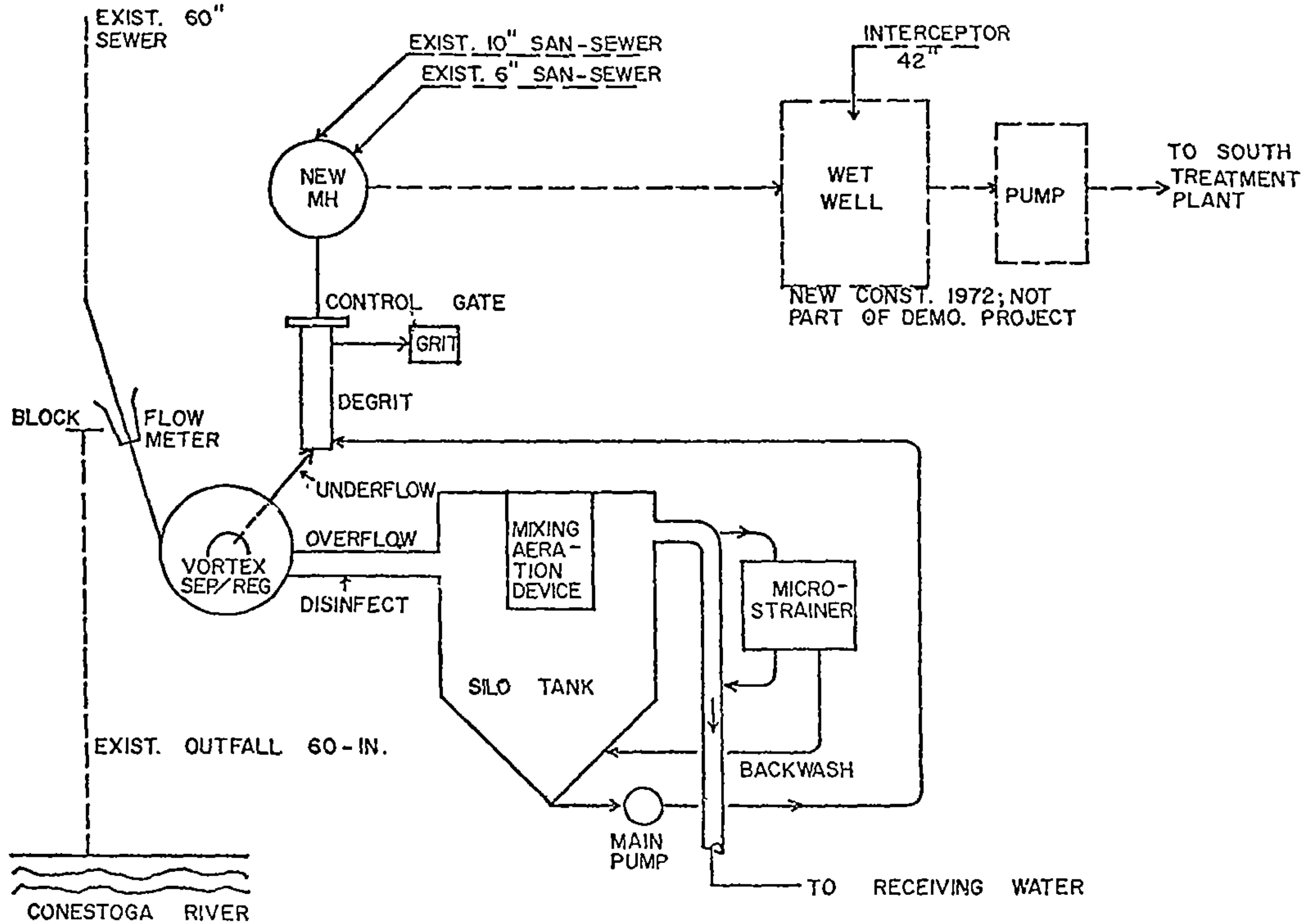


Figure 2-2
Stevens Avenue Drainage Area with Runoff-Transport Numbering System.



PRELIMINARY LANCASTER FLOW DIAGRAM

Figure 2-3
Flow of Treatment-Storage Options at Demonstration Site.

2. DESCRIPTION OF THE STEVENS AVENUE RUNS

A total of four studies comprising nine storms were simulated. The city and its engineers provided input data as well as two overall measurements. The Stevens Avenue district was subdivided into 41 subcatchments. A description of each study and its results are given below:

Study No. 1.--The first study was based on a series of storms between July 29 and August 3, 1971. This six-day period deposited a record amount of precipitation throughout the Lancaster area (variously measured between 7.3 and 9.46 inches). During four of the six days, the storms were very intense over short periods; in one case, being the second heaviest of record. For purposes of simulation, Study No. 1 was divided into six storms. The amount and times of precipitation assumed for each of these six storms are shown in Figures 2-4 through 2-9 and results of computer simulations for each of these storms are shown in the same figures. These figures show the expected quantity and quality of the overflow from the Stevens Avenue district for a given rainfall. These runs indicate that an overflow as high as 400 cfs may be expected for a storm event similar to Storm No. 6.

These computer runs also indicate that total suspended solids and BOD discharges expected in the overflow may be on the order of magnitude of 778 pounds and 635 pounds respectively for Storm No. 5 and 849 pounds and 768 pounds respectively for Storm No. 6. Unfortunately, since actual flow measurements were not taken during this study, it was not possible to determine the actual overflow quantity and quality. However,

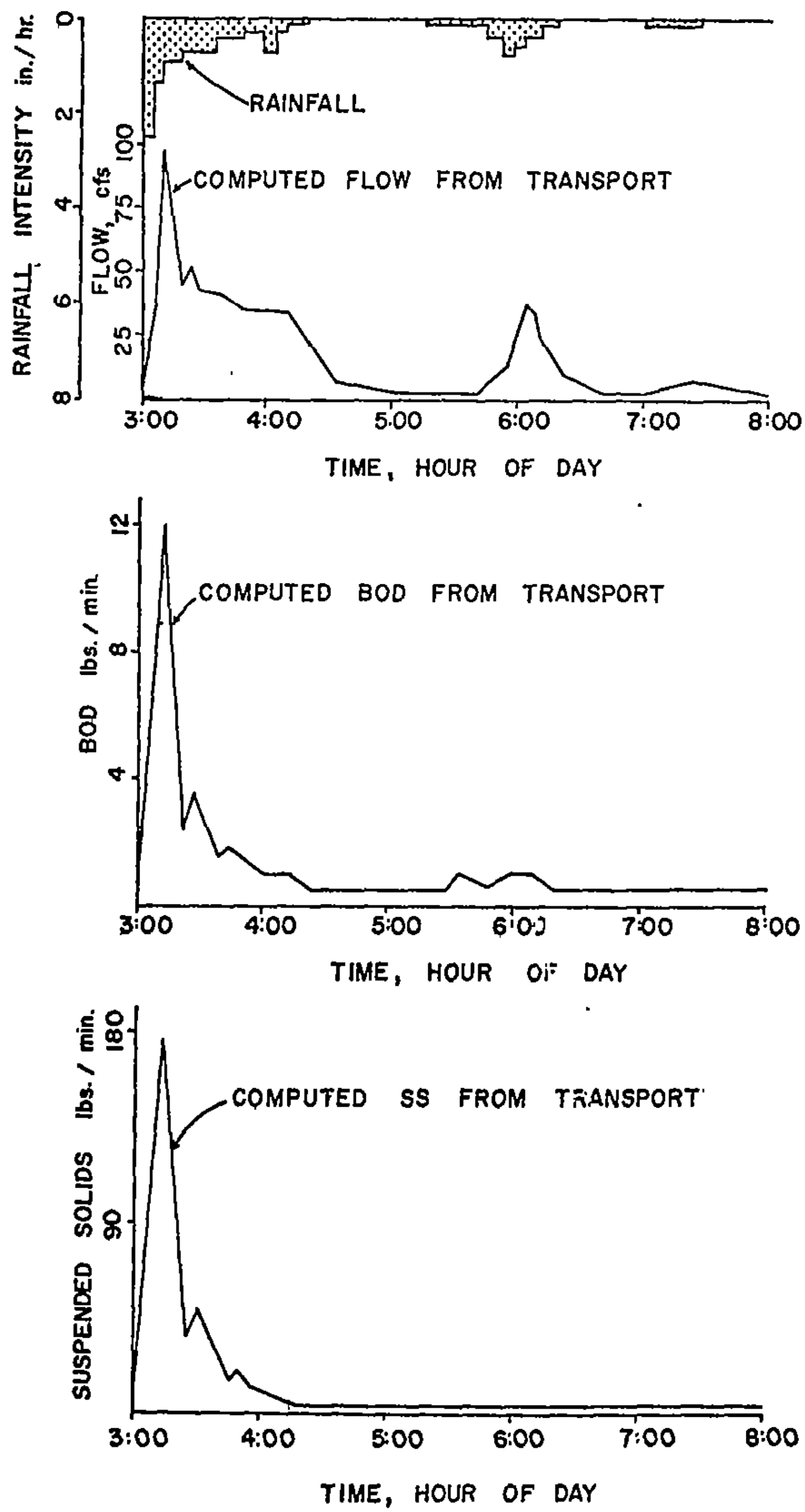


Figure 2-4
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 1.

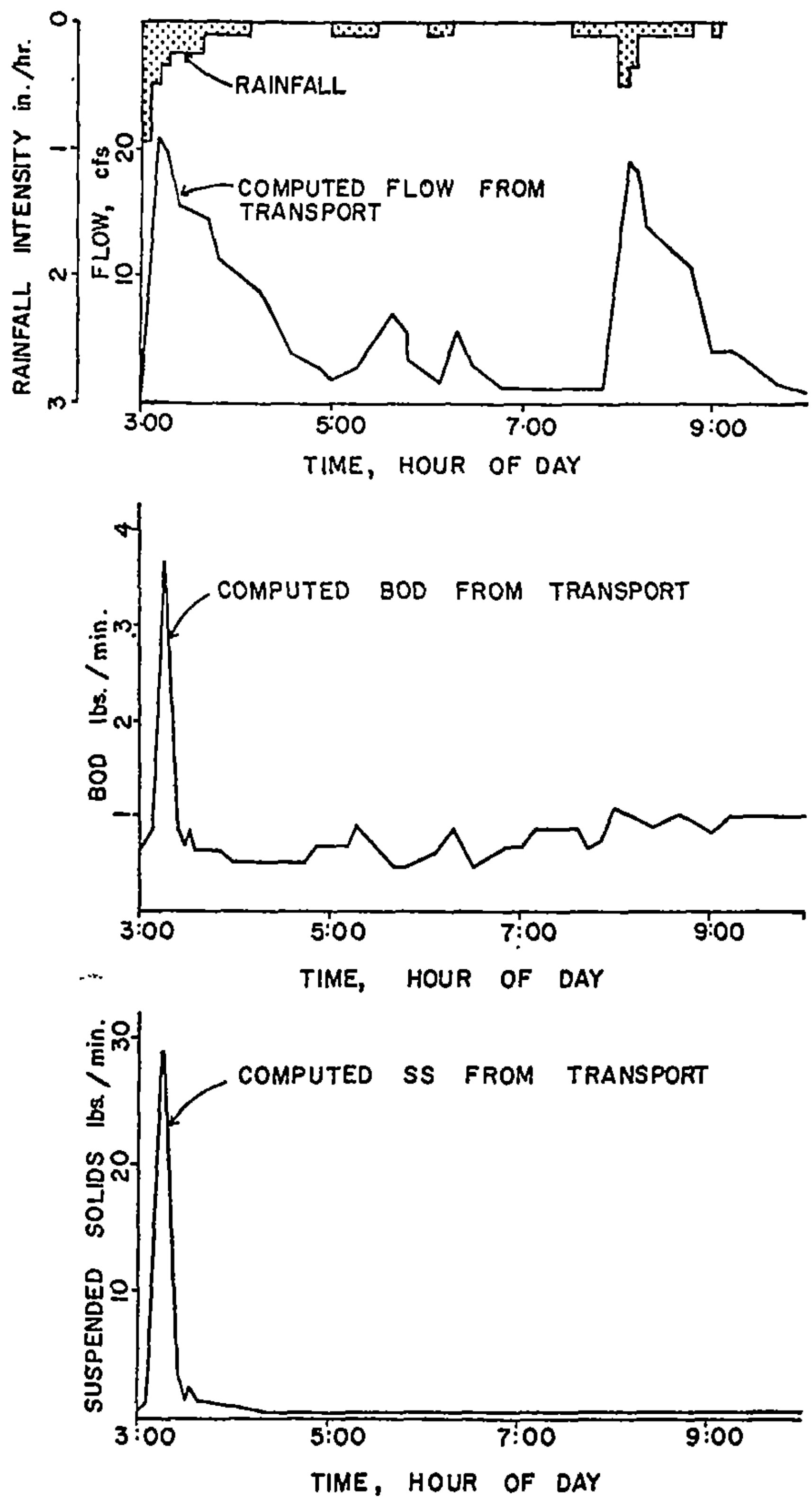


Figure 2-5
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 2.

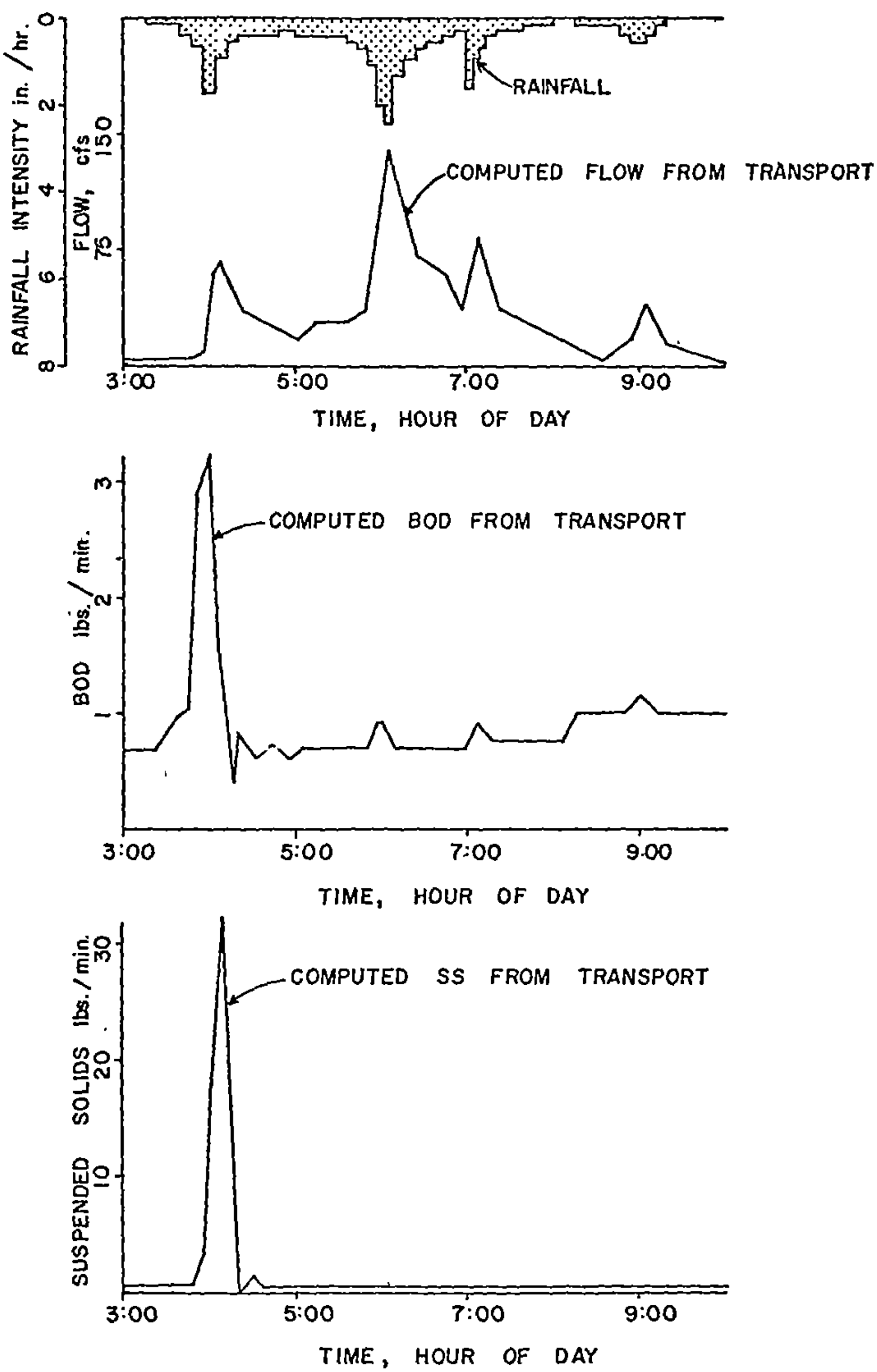


Figure 2-6
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 3.

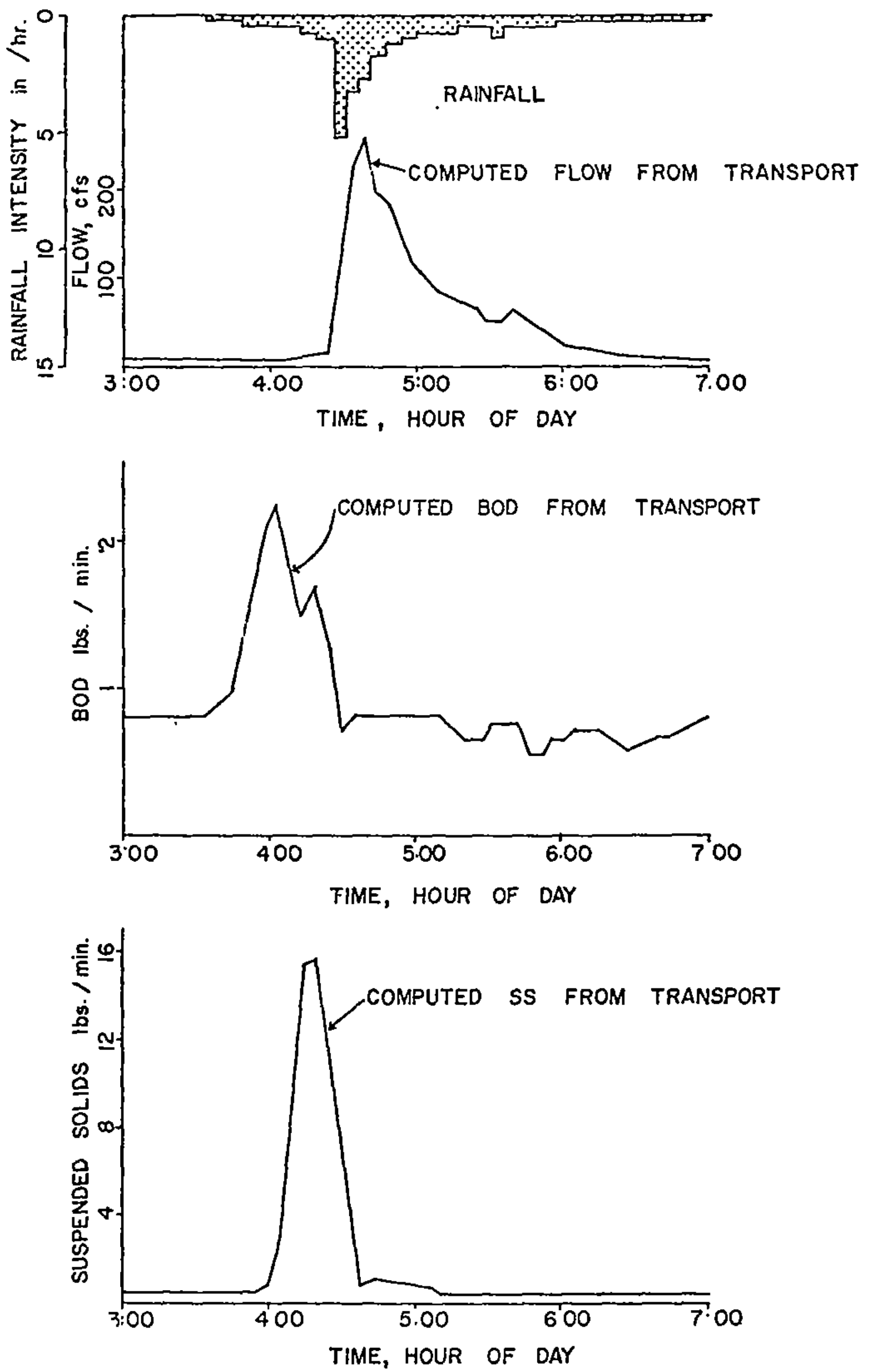


Figure 2-7
Runoff-Transport Simulation for Stevens Avenue.
Study 1, Storm 4.

results of subsequent studies indicate that actual overflows are generally predicted adequately by computer runs. Quality predictions are more variable.

Results of this study were used by APWA in sizing the swirl concentrator. A design flow of this device was established at 150 cfs.

Computer simulation studies were also conducted for all six storms to evaluate the effect of the swirl concentrator-underground silo facilities on the combined overflow quality. The results of Storm Nos. 5 and 6 are shown on Figures 2-8 and 2-9 respectively. As illustrated in these figures, the quality of the overflow is significantly improved through the installation of the swirl concentrator-underground silo.

Study No. 2.--This study consisted of a storm that began in the morning of August 27, 1971 and continued almost 30 hours to the morning of the next day. It resulted in varying amounts of rainfall throughout the city averaging more than 3.5 inches. The results of the computer simulation were similar to those obtained from Study No. 1, and for this reason are not included herein. Again, no measurements were taken during this study.

Study No. 3.--This study is based on a relatively minor rainfall event of March 22, 1972. This study is of special importance, however, because it is one of the types most frequently experienced in terms of intensity of rainfall. It is also one for which relatively complete verification data such as rainfall, flow readings and samples were collected. The rainfall is shown in Figure 2-10 along with results of the computer simulation showing overflow quantity and quality.

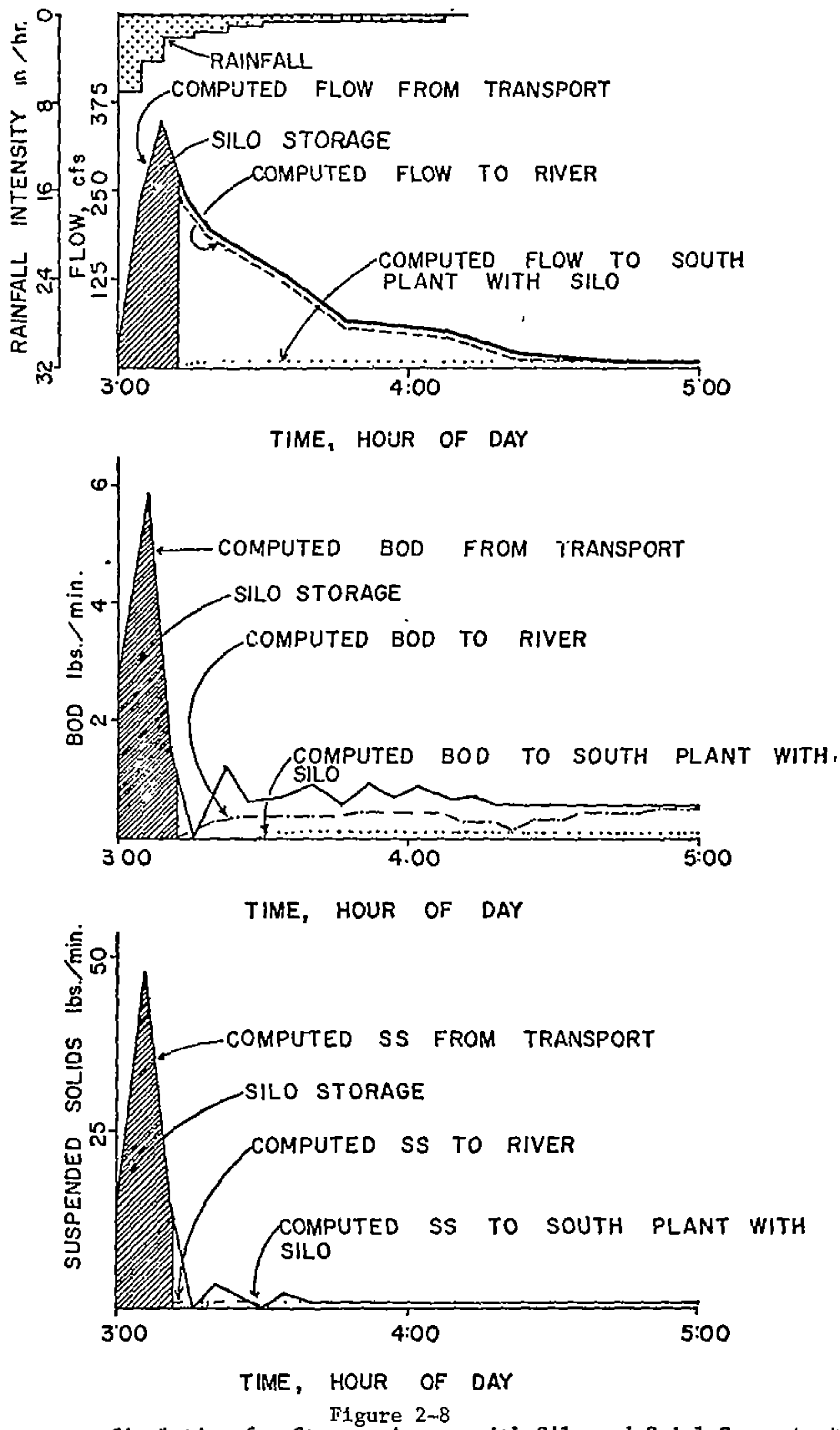


Figure 2-8
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator.
Study 1, Storm 5.

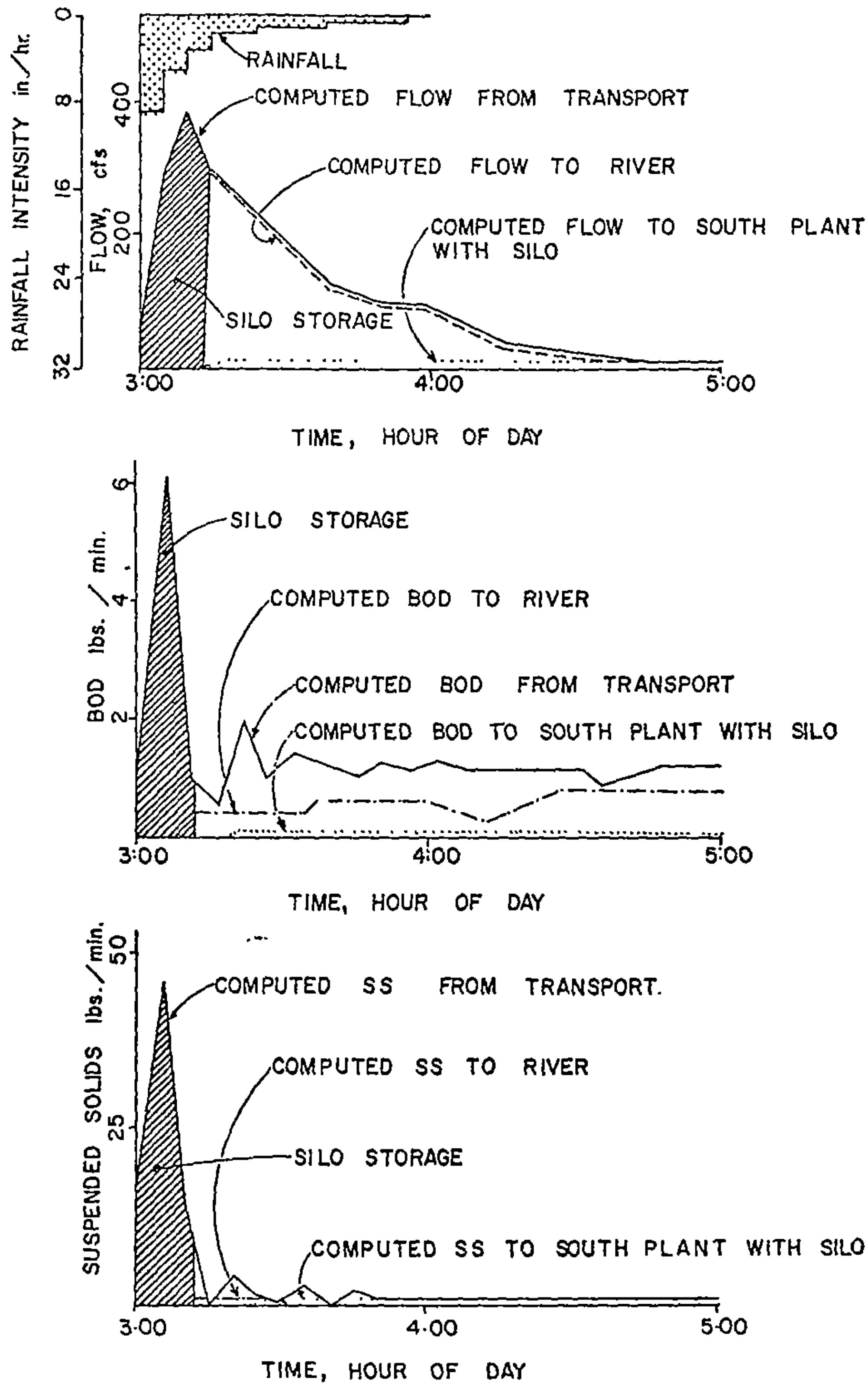


Figure 2-9
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator. Study 1, Storm 6.

Shown in the same illustration are the actual quantity and quality measurements of the overflow. It can be seen that agreement between the computer simulation and the actual measurements of flow is fairly good considering the degree of accuracy of the input data as well as that of the measurements. The agreement between the computed and measured quality parameters is not as good as for flows.

Computer simulations were also conducted on this study to determine the effect of the swirl concentrator-underground silo system. These results are also shown in Figure 2-10. With the silo system, the Model indicates no overflow in the Conestoga Creek.

Study No. 4.--This study is based on a storm that occured on November 29, 1971. This study is also of importance from the standpoint of Model verification as overflow measurements were conducted during this storm. The rainfall and results of the computer simulation for this storm are presented in Figure 2-11 along with the actual measurements for comparison. Again, it can be seen that agreement between the actual measurements and predicted results is fairly good. The predicted results of the swirl concentrator-underground silo system are also shown in Figure 2-11.

3. RUNS IN THE NORTH AND SOUTH DISTRICT

Limited computer simulations were also conducted for the North and South drainage districts. The North district was subdivided into 66 catchments and the South district into 104 catchments. The sewer layouts for the North and South districts are shown in Figures 2-12 and 2-13.

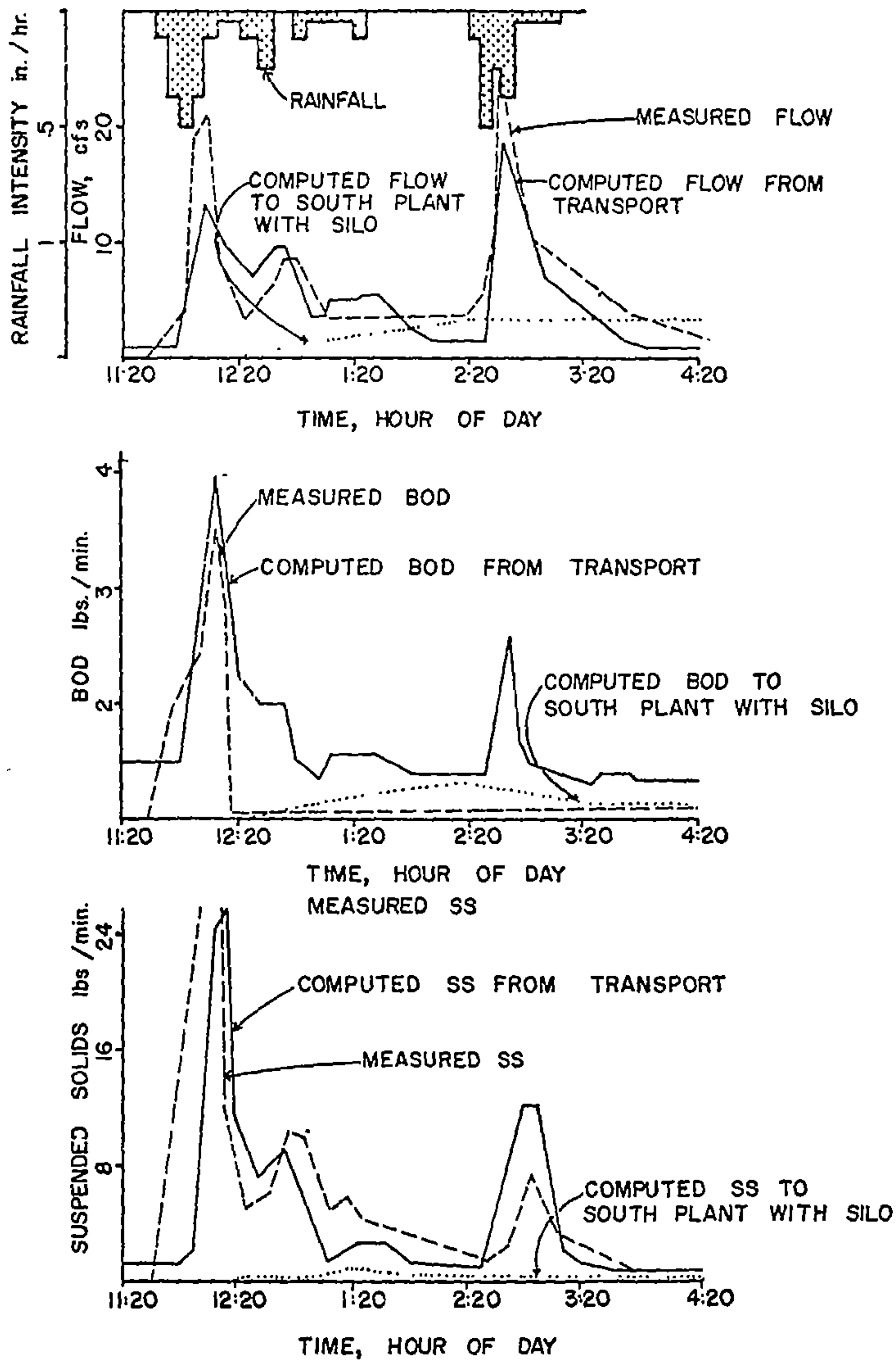


Figure 2-10
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator. Study 3. No Overflow to River Since Silo Capacity not Exceeded.

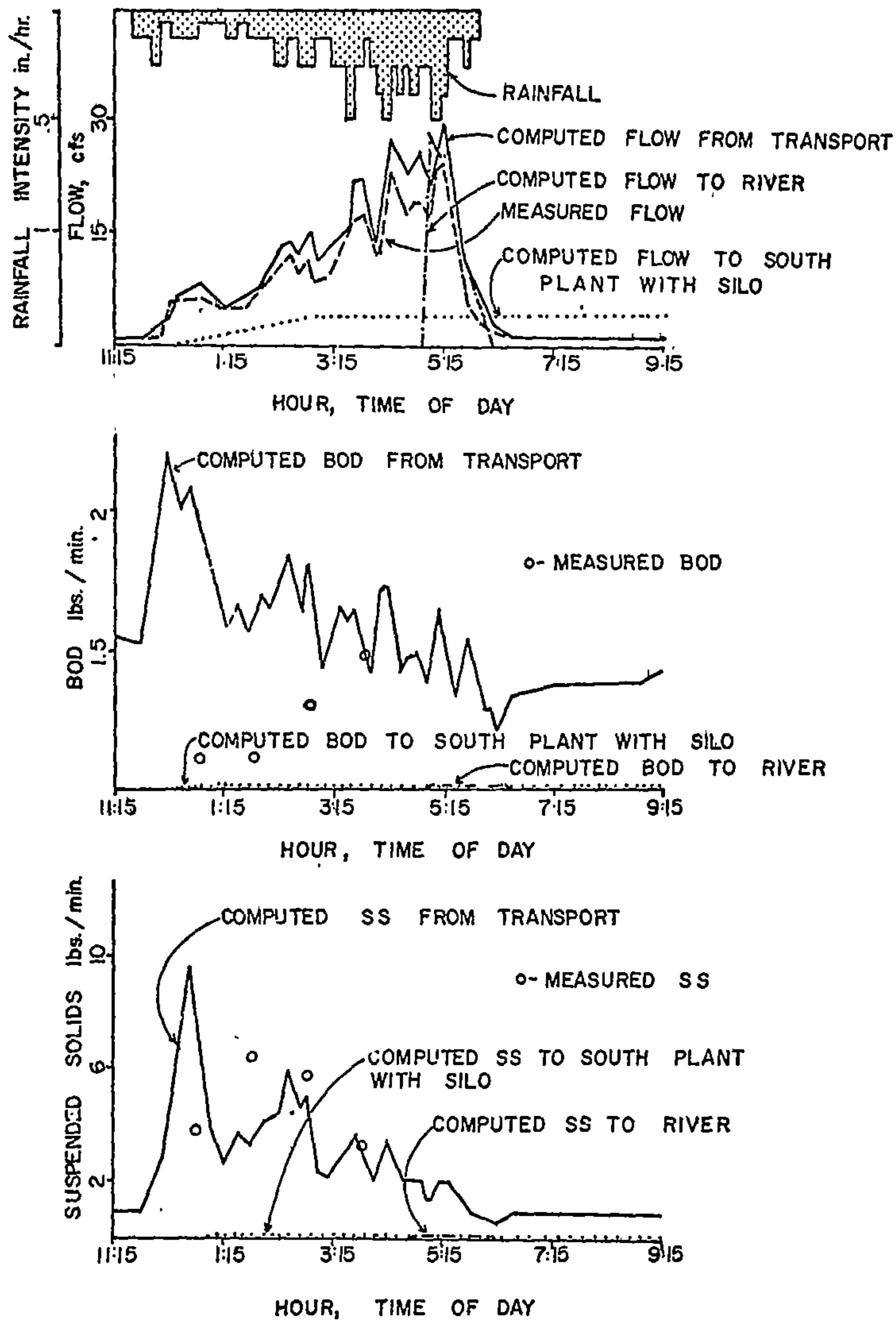


Figure 2-11
Runoff-Transport Simulation for Stevens Avenue with Silo and Swirl Concentrator, Study 4.